Advanced seismic design of a steel fitness centre including a dissipative bracing system

Advanced seismic design of a steel fitness centre building including a dissipative bracing system

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ABSTRACT: A dissipative bracing technology incorporating pressurized fluid viscous spring-dampers is adopted as anti-seismic design strategy of a fitness centre steel building. This represents the first simulated application of the considered protection system, originally conceived and set up for use in multi-story frame structures, to a slender and lightweight building. The characteristics of the spring-dampers and the protective system are recalled, and the performance-based structural design of the building is described. The resulting dimensions of members, as well as the total weights and costs of the steel structure are compared with the ones of a conventional X-shaped bracing solution, complimentarily designed to provide the same performance. A 50% net reduction of costs and an improved aesthetics of the structure, due to the remarkably greater slenderness of the constituting members, come out for the dissipative bracing-based design.

1 INTRODUCTION

The most recent trends in the design of steel buildings favour transparency, lightness and aerial effects, which are obtained by including large glass facades, increasing the free spans of floors and roofs, and implementing innovative architectural shapes and finishes. These emerging trends compel structural designers to reduce the architectural impact of the load-bearing systems further, at the same time meeting the high performance levels required by the last generation of technical Standards on steel constructions. This is especially true for buildings designed in medium-to-high seismicity zones, for which a satisfactory balance between the issues of structural performance and limited architectural impact represents a truly demanding challenge for structural engineers.

Effective solutions are currently offered by the advanced seismic protection technologies available in the market of building industry. Among these technologies, dissipative bracing systems generally provide the most viable solutions for steel buildings, as they visually resemble the bracing strategies traditionally incorporated in steel structural skeletons to absorb wind and seismic loads. This allows keeping substantially unchanged the classical set-ups of steel structures, although drastically reducing the member sections and/or the number of locations where braces are placed, as a consequence of the remarkable reduction of seismic effects produced by the dampers.

Within the wide class of dissipative bracing technologies, a special system incorporating pressurized fluid viscous devices was studied for several years by the second and third author of this paper, also in the frame of international research Projects. Numerical and analytical modelling; experimental characterization and verification; definition of design procedures; and technical implementation of the protective system were particularly carried out within these research activities. Pilot applications were also developed, with special reference to school and apartment buildings.

The study of a novel and more demanding application, concerning a fitness centre and indoor sport steel building in Italy – well representative of the above-mentioned architectural design trends – was ultimately completed. The building is featured by a single span roof, open space interiors, a continuous glass façade, and a perimeter arcade. The architectural design also imposes slender internal and external columns and, in general, very compact structural member sizes. The building is situated in Udine, Friuli region, in North-Eastern Italy. According to the site-dependent seismic classification of the new Italian Technical Standards on constructions (NTC 2008), a medium-high seismicity zone is assigned to the town of Udine and its hinterland.
A summary of the characteristics of the fluid viscous dampers and the protective system, as well as of the previous studies developed on it, is presented in the second section of this paper. A description of the case study building, and a synthesis of the non-linear dynamic design analyses carried out, are offered in the third and fourth section, respectively. A comparison with the dimensions and costs of a traditional anti-seismic solution designed to provide the same performance, complimentarily developed to assess the reductions in member sizes and costs produced by the dissipative bracing technology, is finally illustrated.

2 CHARACTERISTICS OF DAMPERS AND DISSIPATIVE BRACING SYSTEM

The distinguishing mechanical characteristics of the class of silicone fluid viscous dissipaters considered herein (Jarret SL 2008) are represented by: (a) pressurization of the inner casing, produced by a pre-load F_0 applied upon manufacturing, which ensures total self-centering capacity of the devices at the end of their dynamic response; and (b) flow of the silicone fluid through a very narrow annular space between the piston head and the inner casing surface, illustrated in the schematic drawings in Figure 1, which provides a highly non-linear damping capacity (Terenzi 1999, Sorace & Terenzi 2001). Furthermore, the considered devices also act as non-linear elastic springs due to the compressibility of the fluid.

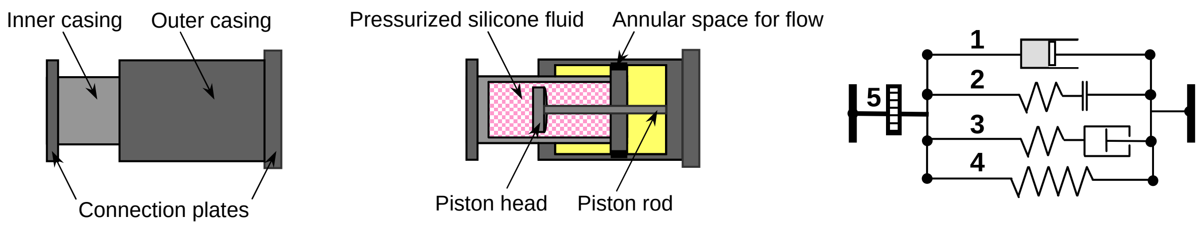

Figure 1. Schematic front view and cross section of a pressurized fluid viscous spring-damper, and relevant computational model (1. non-linear dashpot; 2. gap; 3. hook; 4. non-linear spring; 5. internal force F_0).

The $F_d(t)$ damping and $F_{ne}(t)$ non-linear elastic reaction forces of the fluid viscous spring-dampers can be expressed analytically as follows (Peckan et al. 1995, Sorace & Terenzi 2001):

$$F_d(t) = c\,\mathrm{sgn}(\dot{x}(t))\left|\dot{x}(t)\right|^{\alpha} \tag{1}$$

$$F_{ne}(t) = k_2 x(t) + \frac{(k_1 - k_2)\,x(t)}{\left[1 + \left|\dfrac{k_1 x(t)}{F_0}\right|^{R}\right]^{1/R}} \tag{2}$$

where c = damping coefficient; sgn(·) = signum function; |·| = absolute value; α = fractional exponent, ranging from 0.1 to 0.2; k_1, k_2 = stiffness of the response branches situated below and beyond F_0; and R = integer exponent, set as equal to 5 for pressurized devices (Sorace & Terenzi 2001). An effective simulation of the response of fluid viscous spring-dampers is obtained by combining Eqs. (1) and (2), which are also incorporated in commercial structural analysis programs, such as SAP2000NL (CSI 2008). In addition to a dashpot and a spring, the reaction forces of which are expressed by Eqs. (1) and (2), the computational model of a fluid viscous device is completed by a “gap” and a “hook” assembled in parallel, aimed at disconnecting the device when stressed in tension, and at stopping it when the maximum stroke is reached, respectively (Sorace & Terenzi 2008).

Within this model, also displayed in Figure 1, the static pre-load F_0 is imposed as an internal force to a bar linking the four elements to the interfaced structural members. Further information on this class of dissipaters, including their rate-dependent behaviour and mechanical qualification procedures, can be found in (Sorace & Terenzi 2001, Sorace et al. 2008).
A typical design drawing of a fluid viscous spring-dampers–braces–beam connection of the dissipative bracing system applied in this study is illustrated in Figure 2, showing that the devices are installed, in parallel with the floor beam, at the tip of each couple of supporting steel braces (Sorace & Terenzi 2008, 2009).

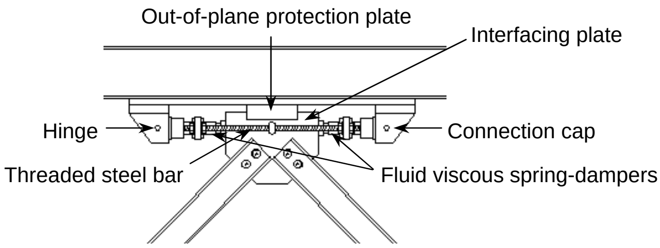

Figure 2. Drawing of a spring-damper–braces–beam connection in a dissipative bracing system.

A half-stroke initial position is imposed to the pistons of both spring-dampers, so as to obtain symmetrical tension-compression response cycles, starting from a compressive-only response of the single devices. This arrangement allows dampers to react immediately also to relatively low dynamic actions, wind included. The half-stroke position is obtained by introducing a pair of threaded steel bars through a central bored plate orthogonal to the interfacing plate of the two devices; the bars are connected at both ends to other two bored plates, threaded into the external casing of the spring-dampers. The axial force required to drive the pistons at their half-strokes is applied to the steel bars by a torque wrench, by acting on the nuts in contact with the two plates threaded on the devices. The terminal section of the external casing of each fluid viscous device is encapsulated into a steel “cap” hinged to two vertical trapezoidal plates welded to the upper horizontal plate of the assembly, which is fixed to the lower face of the floor beam. The caps constrain the spring-dampers to move with the same displacement as the floor beam. Thanks to the rigid support offered by the diagonal braces, mounted in an inverse-chevron configuration, this installation helps the devices exploit the entire interstory drift between the upper and lower floor across which they are placed. In order to prevent out-of-plane displacements of the system, two small protection plates are mounted in parallel to the vertical plate interfacing the devices, at 1 mm distance from its faces. A Teflon disk is placed on the inner faces of the protection plates, to avoid any frictional effects in case of accidental contact with the interfacing plate during seismic response.
Extensive experimental activities were carried out on the dissipative bracing technology (Sorace & Terenzi 2003, 2004, 2008, Molina et al. 2004), with a view to studying its practical application to the new design and the rehabilitation of steel and reinforced concrete buildings. They evidenced that high seismic performance levels of tested structures are always attained in protected conditions. Moreover, the experiments allowed developing a thorough calibration of the formulated analytical and numerical models, as well as of the proposed design methodology of the system (Sorace & Terenzi 2003, 2008).

3 DESIGN CASE STUDY

The design case study examined herein was aimed at exploring new fields of application of the dissipative bracing system incorporating fluid viscous devices, with special regard to slender steel structures. The building is conceived for use as a fitness and indoor sport centre, with a rectangular plan and a complete perimeter arcade. The external dimensions are (60×30) m×m in plan, with a height of 18 m. The arcade is 3 m-wide, which determines net internal dimensions of (54×24) m×m. An intermediate floor is situated at a height of 6 m on one side, covering a reduced area in plan, equal to (20×24) m×m. Two global architectural renderings of the building, and three views of the interiors, are reproduced in Figures 3 and 4, respectively. These images show the appearance of the structure in the case of a conventional bracing solution, which is compared in the final section of the paper with the dissipative bracing configuration adopted in this design. Views of a typical joint of the roof and floor trusses, made of tubular profiles, and a connection of a truss to relevant column and longitudinal beams, are displayed in Figure 5. The shapes and dimensions of the trusses and the secondary elements of roof and floor are identical for both bracing configurations, as their design is governed essentially by gravitational loads, rather than by seismic action.

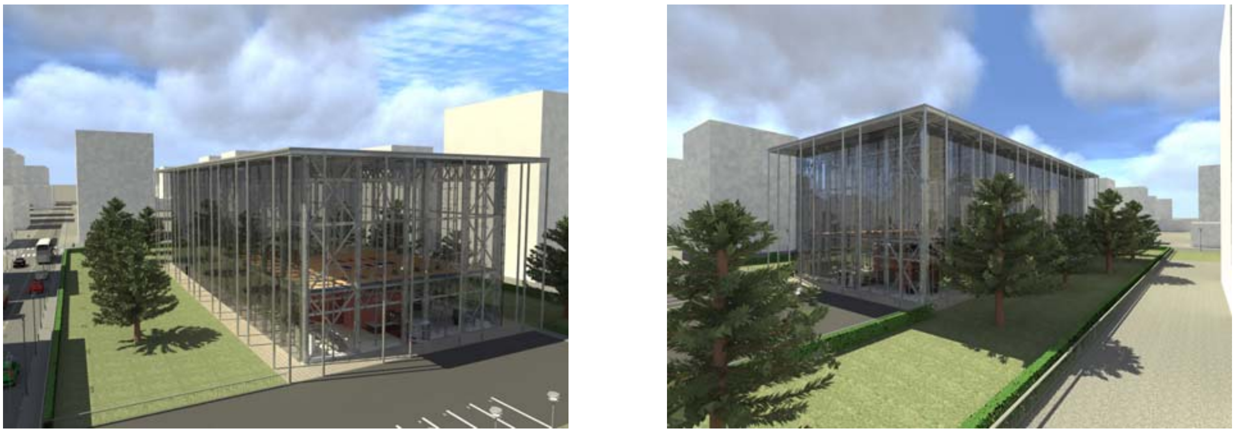

Figure 3. External architectural renderings of the building.

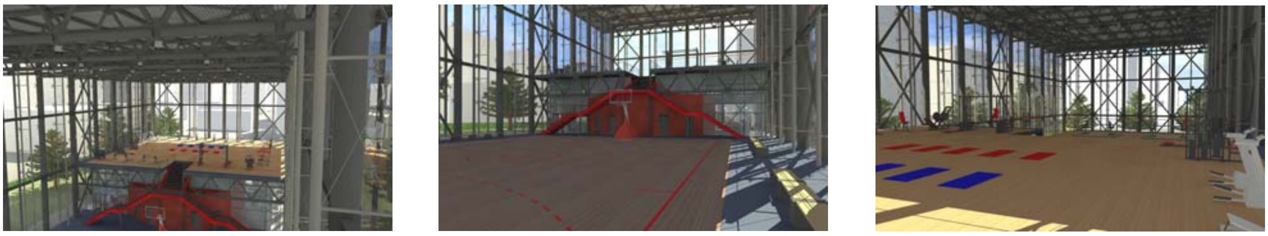

Figure 4. Internal architectural renderings of the building.

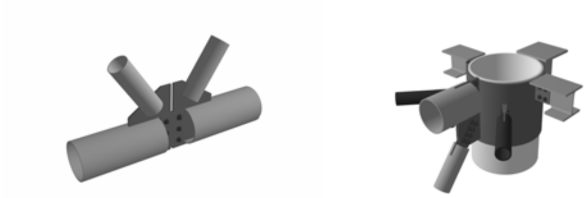

Figure 5. Views of a typical joint of roof and floor trusses, and a truss-column-longitudinal beams connection.

4 DESIGN AND PERFORMANCE EVALUATION ANALYSES

The preliminary design of the dissipative bracing system was developed by the general criterion formulated for applications to frame structures in Sorace & Terenzi (2008), herein adapted to the special configuration of the building, which is not featured by a classical frame-like seismic response, as it does not include complete intermediate floors. The non-linear dynamic design verification and performance evaluation analyses were carried out by the finite element model shown in Figure 6, already incorporating the protective system, elaborated by the above-mentioned SAP2000NL program (CSI 2008). A set of five artificial accelerograms generated from the elastic response spectrum of NTC (2008) was assumed as input to the analyses, by scaling the peak ground amplitude to the values referred to the different design performance levels discussed below.
Two types of fluid viscous spring-dampers were adopted, namely BC1FN and BC1GN type, selected from the basic catalogue of the manufacturer (Jarret SL 2008). Their mechanical characteristics are: nominal energy dissipation capacity E_n = 60 kJ (BC1FN), 120 kJ (BC1GN); maximum reaction force R_{max} = 150 kN (BC1FN), 220 kN (BC1GN); and stroke d_{max} = 65 mm (BC1FN), 80 mm (BC1GN). The devices were placed over two vertical alignments along both directions in plan, for a total of 24 pairs of elements, as sketched in Figure 7. BC1GN dissipaters were installed in the longitudinal direction, and BC1FN dissipaters in the transversal one, except for the first level on the

intermediate floor side, where BC1GN elements were incorporated. This difference between the two transversal sides is explained by the need of compensating the torsion effects caused by the eccentric position of the floor in plan. A detailed rendering of the installation of a pair of devices is displayed in Figure 8.

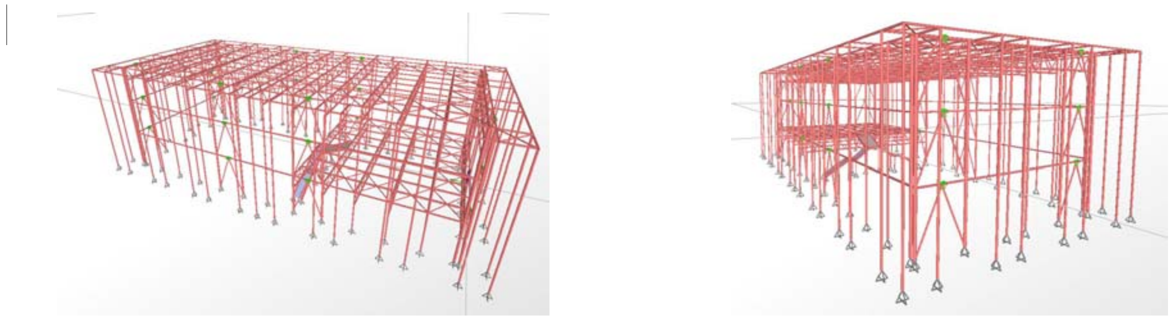

Figure 6. Views of the finite element model of case study structure in the presence of the dissipative bracing system.

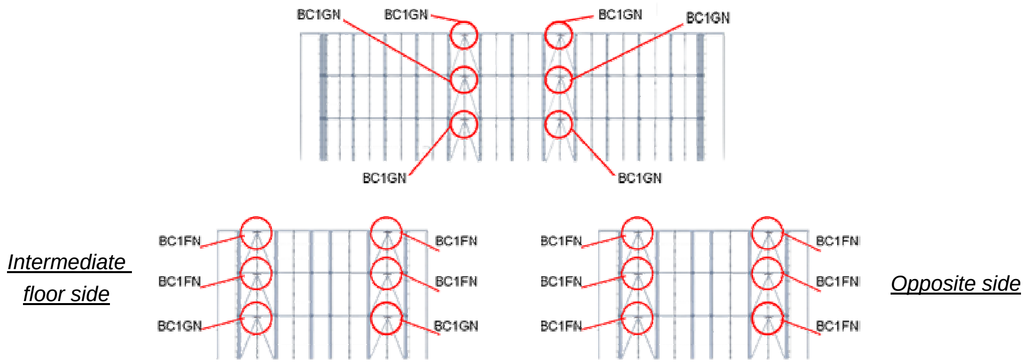

Figure 7. Placement of spring-dampers along the longitudinal (upper image) and transversal directions.

Figure 8. Rendering of the installation of a pair of fluid viscous spring-dampers.

Similarly to the most prominent Standards worldwide, four performance levels are considered in the seismic design section of the new Italian Technical Standards (NTC 2008), that is, Operational, Immediate Occupancy, Life Safety, and Collapse Prevention levels. According to the revised seismic classification of the town of Udine, and the public use of the building, the following amplitudes are assigned to the peak ground accelerations for the analyses referred to the four performance levels: 0,08 g, 0,11 g, 0,28 g, and 0,35 g. Within this case study, unified design objectives were assumed for the first three levels, which consisted in targeting no damage to all the structural and non-structural components of the building. As a consequence, the verifications for the Life Safety level automatically satisfied also those relevant to the two lower levels. The most demanding request of performance concerned the highly vulnerable glass façades, which represent the most costly construction elements of the building. In order to avoid damage to the façades, the relative displacements between adjacent panels should be limited within around 1 mm. This limitation was also extended to the Collapse Prevention performance level, and substantially governed the design of the dissipative bracing system, as well as of the remaining members of the earthquake resistant steel skeleton, that is, columns and longitudinal beams. Indeed, the dimensions determined by the attainment of the non-structural performance objective above, also allowed matching the requirement of no damage (i.e., total elastic response) of structural members. The study of the response of glass façades was carried out by separate finite element models, where the maximum displacements deduced from the analysis of the main structure were imposed to assess the achievement of the limits postulated for the panels. A global view of the model of a longitudinal glass façade, a deformed shape resulting from computation, and a zoomed view on a vertical alignment, are displayed in Figure 9.

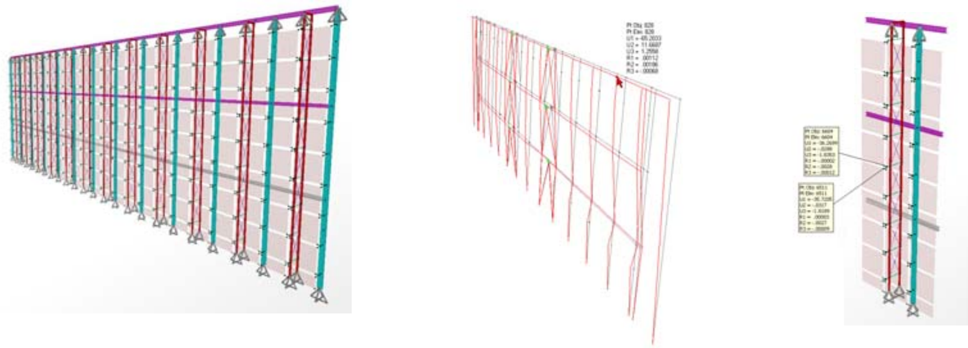

Figure 9. Finite element model of a glass façade, deformed shape derived from computation, and zoomed view on a vertical alignment.

The response of the steel structure obtained from the analyses referred to the Life Safety limit state is synthesized in Figures 10 through 13. Figure 10 shows a schematic roof plan and the positions of the upper joints of the two most distant internal columns, which were assumed as control points for the evaluation of the global torsion effects of the building. The graphs in Figure 11 highlight a substantial superimposition of the displacement response time-histories of these joints, derived from the application of the most demanding among the five input accelerograms. These data confirm that the adopted layout of the protection system allows effectively constraining global torsion effects on the building. The force-displacement response cycles of the most stressed pairs of BC1FN and BC1GN devices, obtained from the same input ground motion, are demonstratively plotted in Figure 12. The energy time-histories reported in Figure 13 show that around 90% of total input energy is absorbed by dissipaters, for both directions in plan, as targeted in the design of the system.

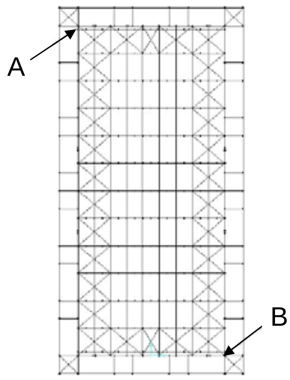

Figure 10. Schematic roof plan and control joints of torsion response.

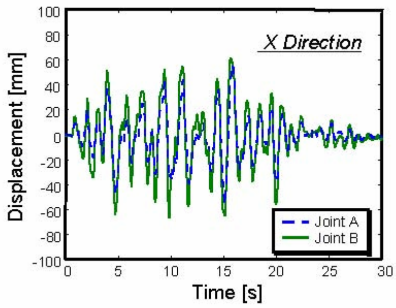

Figure 11. Displacement time-histories of joints A and B obtained from the most demanding input accelerogram in the analysis at the Life Safety performance level.

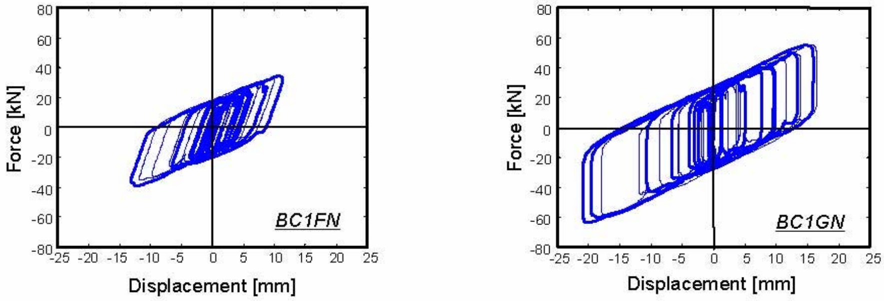

Figure 12. Response cycles of the most stressed BC1FN and BC1GN pairs of spring-dampers obtained from the same accelerogram referred to in Figure 11.

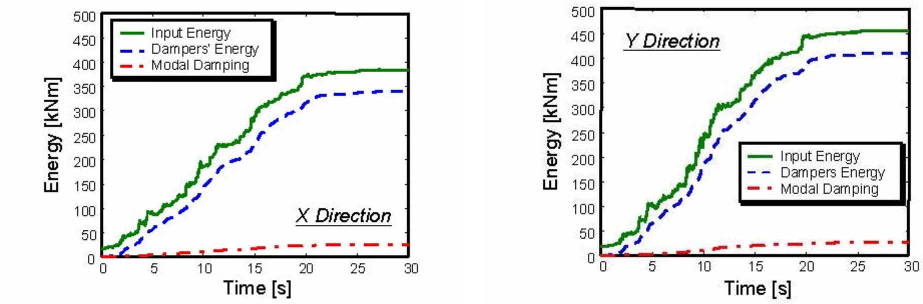

Figure 13. Energy time-histories in transversal (X) and longitudinal (Y) direction obtained from the same accelerogram referred to in Figures 11 and 12.

Concerning the Collapse Prevention performance level, in addition to the requirement of null or very limited damage to structural and non-structural components postulated even for this limit state, a control was also developed on the correct operation of spring-dampers. This was assessed by surveying the maximum displacements of each device, which resulted to be lower than the available stroke for both types of dissipaters installed.

5 COMPARISON WITH A TRADITIONAL BRACING SOLUTION

An alternative design of the building was carried out by using a conventional non dissipative X-shaped bracing solution, with the aim of establishing a comparison with the advanced anti-seismic strategy discussed in the previous sections. The conventional design pursued the objective of reaching the same global performance ensured by the dissipative bracing system, to determine a mutual reference for a direct comparison of member dimensions, as well as of total weights and costs of the steel structure. The resulting sections of the members of the earthquake-resistant skeleton are listed in Table 1 for the two designs. As observed above, trusses and secondary elements of roof and floor are not included in this Table, as their dimensions are determined by gravitational loads, and thus are identical in the two cases. The same holds true for the columns of the arcade.

Table 1. Structural member sections for the dissipative and conventional bracing design solutions

Member types	Dissipative bracing	Conventional bracing
Tubular profiles (diameter – mm)		
Internal columns – type 1	323.9	610
Internal columns – type 2	219.1	508
Vertical braces	168.3	273
Italian H-shaped profiles		
Longitudinal and transversal beams	HEA 160	HEB 300/320

The total weights and estimated costs of the structure for the two design hypotheses are recapitulated in Table 2 (in the case of the dissipative bracing solution the costs of spring-dampers are already included in the amount reported in this Table). These data highlight a 60% reduction in weights, and 50% in costs, for the dissipative bracing-based design. In addition to these remarkable economic advantages, the drastic drop in member sections and weighs leads to a considerably lower impact on the visual perception of the building. This is demonstratively shown by the renderings in

Figure 14 for a single mesh of braces. The aesthetical benefits are evident, especially for what concerns columns, extremely massive in the case of the traditional bracing design.

Table 2. Total weights and costs of the dissipative and conventional bracing design solutions

Weights and costs	Dissipative bracing	Conventional bracing
Weights (tons)	250	630
Costs (euros)	650,000	1,300,000

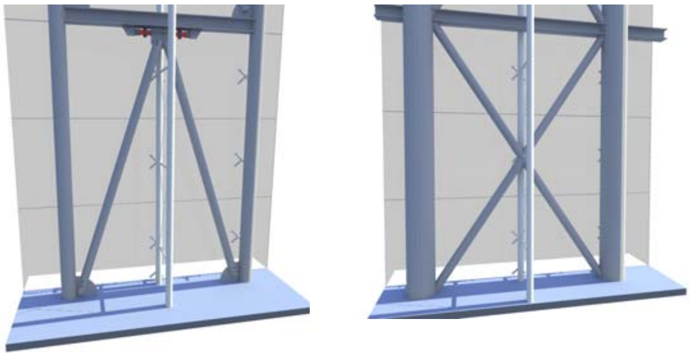

Figure 14. Rendering of a single mesh of braces for the two design solutions.

By summing up the results of this explorative design, a first positive indication is obtained about the application of dissipative bracing technologies also to the seismic protection of lightweight structures featuring the latest generation of steel architectures.

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